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STUDIES IN DRIVER BEHAVIOUR, WITH APPLICATIONS
IN TRAFFIC DESIGN AND PLANNING. TWO EXAMPLES



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STUDIES IN DRIVER BEHAVIOUR
WITH APPLICATIONS IN
TRAFFIC DESIGN AND PLANNING

av

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STUDIES IN DRIVER BEHAVIOUR WITH APPLICATIONS IN TRAFFIC DESIGN
AND PLANNING. TWO EXAMPLES

ARNE HANSSON





FOREWORD

In this report are summarized two research projects, which were conducted separately during the years 1972 - 1974.

1. *BILISTERS VÄGVAL I TÄTORTER*
(Route choice in urban areas)
Trafikteknik LTH Bulletin 10 1975 (in Swedish)
2. *TRAFIKAVVECKLING I KORSNINGAR UTAN SIGNALREGLERING*
(Traffic flow at uncontrolled intersections)
Trafikteknik LTH Bulletin 11 1975 (in Swedish)

The first study was sponsored by the Swedish Transport Research Delegation and carried out at the University of Lund under the guidance of Professor Gösta Lindhagen. The second study is part of a major project concerning road and intersection capacities, undertaken by the Swedish National Road Administration and with Professor Stig Nordqvist, VBB Vattenbyggnadsbyrån, as investigator.

Acknowledgments are also due to several others, who all contributed in different ways to the studies mentioned and to the publication of this report. The author regrets that they are too numerous to be mentioned individually.

Lund in November 1975

Arne Hansson

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<i>TRAFIKAVVECKLING I KORSNINGAR UTAN SIGNALREGLERING</i> (TRAFFIC FLOW AT UNCONTROLLED INTERSECTIONS) See LTH Bulletin 11 1975	

LIST OF SYMBOLS

Symbols used in Chapter 2

P_i	probability of choosing alternative route i
c_i	route resistance, averaged over individuals
C_i	route resistance (a stochastic variable)
x_{ij}	characteristic j for alternative route i
X_{ij}	x_{ij} as perceived by an individual (a stochastic variable)
a_j	weight of characteristic j in route resistance, averaged over individuals
A_j	like a_j but as conceived by an individual (a stochastic variable)

Symbols used in Chapter 3

a	critical head-way	a_l = critical lay a_g = critical gap a_f = move-up time
b	load factor	
c	coefficient of variation	
d	delay	d_b = service-time d_f = running delay d_k = queueing-time d_v = waiting-time
h	head-way	h_u = head-way in minor road flow h_o = head-way in major road flow
K	capacity	
N	number of vehicles in queue	
q	flow	q_u = minor road flow q_o = major road flow

SUMMARY

In this report are briefly described two projects, both of which apply studies of individual road user behaviour to build predictive models for use in traffic design and planning.

The first study concerns route - choice behaviour in urban areas, with applications in, for example, traffic assignment models. A theoretical model is considered, which describes route-choice in terms of the road users' preferences for various route characteristics and of their estimates of the actual values of such characteristics. Using data from 691 home interviews, different hypotheses concerning the structure of such a model are tested and some parameter estimates obtained.

The second study was undertaken as part of a major investigation concerning capacity of roads and intersections and focus on intersections without signal control. Theoretical models for the calculation of capacity and road user delay at such intersections are reviewed. Most important parameter in these models is the headway required in a major road flow for a waiting minor road vehicle to pass (critical headway). To study this parameter, and its dependence on various influencing variables, a field-study is undertaken including 18 intersections. In general the results agree well with those of previous studies.

INTRODUCTION: TRAFFIC PLANNING, DESIGN AND INDIVIDUAL BEHAVIOUR

The purpose of traffic planning and design is to achieve a transportation system that admits a high mobility for persons and goods, at a low national cost and with a minimum of disturbances.

For practical planning, this general objective has to be substituted by operational efficiency measures. Ideally, these measures form a hierarchical structure with an increasing degree of specification as the implementation of the plan approaches and as more information becomes available. At some intermediate level, for example the criteria may be defined in terms of construction costs, traffic safety, level of service, environmental effects etcetera.

In order to evaluate such criteria for planning purposes, a traffic forecast is most often needed. A usual procedure, for the estimation of future traffic in an urban area, contains the following steps:

- (I) Trip generation: calculating the expected amount of trips from future land use data, expected car density etc
- (II) Trip distribution: estimating the distribution of generated trips, resulting in a zone-to - zone trip matrix
- (III) Modal split: estimating the distribution of zone - to - zone trips by different means of transport
- (IV) Traffic assignment: estimating the distribution of zone - to - zone trips (by car) on alternative routes

The procedure may be supplied with feed-backs, allowing for the fact that the assigned traffic flows affect the level of service. In turn, this affects the attraction of the routes used and eventually the modal and zonal distributions. Commonly, however, the four steps are treated as independent, supposedly reflecting four sequential choice situations facing a potential trip-maker.

Conventional models of this type, for example the so called gravity models, are semi-empirical macro-models using aggregated data and based on rather crude assumptions concerning the behavioural process. As major deficiencies with these models have been mentioned (1) their difficulty to cope with radical changes in a transportation system, such as the introduction of a new mode of transport (2) their inability to admit evaluation of benefit separately for different socio-economic categories, and (3) their inaccuracy when dealing with other problems than over-all forecasts, which is due to inconsistencies in the internal structure.

As transportation can be considered a link between needs and satisfaction of needs, it seems obvious that travel patterns can be "explained" only in a larger behavioural context, by an analysis of subjective perceptions and evaluations. The "choice", of a destination, a mode of transport or a route, is the observable end of a behavioural process.

Recognizing this, several attempts have been made to develop disaggregated, behavioural choice models for the purpose of traffic forecasting and the evaluation of transport systems. Most of these attempts focus on one of the sub-models in the four-step procedure described, and most of these on the choice of mode (III).

It is hypothesized that the sub-models developed may overcome some of the deficiencies with the conventional models: (1) since the parameters are more related to basic human needs and desires, and less to temporary technological innovations, they are more likely to remain stable through time ; (2) the models are based on the behaviour of individuals and thus allow aggregation according to any variables desired; and (3) their increased internal consistency makes possible a more flexible choice of input-output relations.

In Chapter 2 of this report is described an attempt to develop a behavioural choice model concerning the fourth stage in the forecast procedure, the route selection. The study proceeds in the steps of previous works, mainly concerning modal choice. For route selection, however, some additional difficulties occur: the number of possible routes are usually many more than the number of possible modes, and different routes much more similar and less well-defined than different modes. It is also recognized that the study described here is

but a very small step towards an understanding of the underlying behavioural precesses.

The concepts "Planning" and "design" imply a whole range of activities, with different horizons in time and space: comprehensive urban or regional planning, small area planning and area traffic control, localization of new facilities, reconstruction of existing facilities etcetera. The choice of location for a new highway, for example, might still require some of the steps in the forecast model: different locations could affect the distribution of traffic on different roads, which might change the cost/benefit-ratio. A route choice model would be needed to predict this effect.

The difference between "design" and "planning" is thus one of grade rather than one of kind. The criteria is more specific, the available information more detailed and the number of possible solutions perhaps fewer, but the object ought still to be the evaluation of all these possible solutions against the criteria defined.

Either this evaluation is performed for the individual design project, or with a lower frequency by some authority before issuing recommendations and norms, a method is needed for estimating the efficiency measures contained in the criteria. For road traffic, such measures may include expected travel times or delays and other aspects of convenience, expected accident frequencies and environmental ill-effects or, as is often the case in urban areas, possible capacity and required storage space of queues.

For the prediction of these measures, models are needed that relate delays, accident frequencies etcetera to the design variables and to the traffic flows expected. Since the object is to find an optimal solution, these models should ideally have enough internal consistency to be able to represent the effects to individual input variables. To achieve this, the models ought to be based on a correct description of road user behaviour.

In most cases though, and for most of the efficiency measures mentioned, this behaviour is too complex to allow the detailed description required - at least in the form of diagrams or mathematical formulas. The behavioural model must then be substituted with empirical, non-explanatory relationships, however incomplete.

One of the few exceptions could be the problem treated in Chapter 3 of this report, that of predicting capacity, delay and queue lengths at unsignalized intersections. The relevant behavioural process in this case is rather simple and can (with a rough simplification) be expressed by a single parameter: the headway needed in the flow on the major road for a waiting car to cross. The report describes an attempt to measure this parameter, and discusses some theories for the calculation of capacity, delay and queue lengths.

2. EXAMPLE 2: ROUTE CHOICE IN URBAN AREAS

2.1 Background and purpose

In traffic forecasts for urban areas, the assignment phase often contributes with a major part of the inaccuracy in the final results as well as with a considerable share of the costs. It is a reasonable hypothesis that the inaccuracy mentioned, often yielding errors of a systematic nature, is due to erratic or too coarse assumptions (implicit or explicit) regarding route choice behaviour.

Other applications may concern planning in smaller scales or area traffic control. Here, the problem could often be to estimate the redistribution of traffic that will result from certain measures. For such tasks, as well as for applications to future traffic systems of yet non-existing kinds, the models used need an explanatory power that has not yet been achieved.

In order to meet such demands a route selection model has to consider the reasons for a particular choice, that is it must be based on a theory for individual choice behaviour. Predictive models or sub-models with such a scope, within the field of traffic planning, have been underway for some time but particularly concerning the choice of mode. The purpose of this study is to investigate and test the possibility to formulate such individual choice behaviour models for route selection, and if possible to obtain some parameter values for future applications.

2.2 A theoretical approach

Previous route-choice studies

The results of several earlier route-choice studies, concerning the effects of various factors, can briefly be summarized as follows:

- most important route characteristics are travel times and, eventually, route lengths

- several other characteristics may be of importance, for example variables related to "driving comfort", and big deviations are still left unexplained
- the fact that different drivers choose different routes in similar situations may to a large extent depend on the fact that, due to incomplete knowledge, they estimate (for example) the respective travel times differently
- some first attempts in using attitudinal data indicate that individual differences exist in the evaluation of route characteristics, and that these differences affect route choice.
- difficulties are encountered when specifying models that consider more than two alternative routes, and some results indicate that the commonly used hypothesis

$$P_i = U_i / \sum_j U_j \quad P_i = \text{probability of choosing route } i$$

$U_i = \text{"attraction" of route } i$

is insufficient

- difficulties also arise when defining the concept of "alternative", in cases where the routes considered overlap in different ways

The results mentioned were utilized in formulating the hypotheses for this study.

Requirements of a route-choice model

Necessary or desirable properties of a predictive model for route-choice can be derived both from point of view of the prospective applications and from certain theoretical considerations.

In view of the applications, it is required that the input of the model only consists of such data, that realistically can be considered available in the planning process. Alternatively, it should be possible to reduce the model to such a form by suitable approximations. The output of the model should be such data that is relevant to the planning

situation, which might include not only estimated traffic volumes but also assigned volumes of different categories of road users (such as, for example, different trip purposes).

The theoretical considerations pertain more to the accuracy of the model and considerably limit the amount of possible analytical formulations. One requirement postulated, for example, consider the continuity with respect to the definition of "alternative route": there must be no arbitrariness in the model due to the enumeration of the alternatives.

A model for individual choice behaviour

Every individual can be supposed to choose that alternative, which seems least unfavourable to him. Suppose that this "degree of unfavourability" can be classified along a continuous scale, by a variable C . This C shall be called "resistance" here and is equivalent to the "utility" (with opposite sign) in economic theory.

Considering several routes $i = 1, 2 \dots$ and several individuals, the different C_i will denote a set of stochastic variables. Assume that different individuals choose independently, which is true as long as the traffic flows assigned are marginal only. By definition of C , route i will receive a share

$$P_i = P(C_i < C_j \quad ; \quad j = 1, 2 \dots ; \quad j \neq i) \quad \dots (2.1)$$

The distributions of the C_i 's must pertain to

- (1) the characteristics of the respective routes, i.e. travel times, route lengths etcetera, or to the estimates of these characteristics by the road-users.
- (2) the relative "weights" that the road-users attach to these characteristics, when (subconsciously) evaluating the resistances

A simple form for C_i could be, for example

$$C_i = \sum_k A_k X_{ik} \quad \dots(2:1)$$

where index k denotes a certain characteristic (such as travel time), X_{ik} the estimate of this characteristic for alternative i and A_k the weight of the characteristic k .

Different assumptions concerning the statistical distributions of the A_k 's and the X_{ik} 's, either one of sets eventually assumed to be a set of constants, will lead to different models. Most of these models have been used in studies of modal-split, though usually for only two alternatives.

If the A_k 's are assumed to be stochastic, correlations between the resistances for different alternatives are implied, which considerably complicates the model for predictive applications. It is hypothesized that the variance in the weights can be explained satisfactorily by input variables such as trip-purpose. If this is the case, the model (2:1) - (2:2) can be applied separately to different strata with a constant set of A_k 's within each strata.

However, the resistances C_i 's will in general still be dependent due to correlations between the estimates X_{ik} for different alternatives. It is proposed that this correlation between an X_{ik} and an X_{jk} is proportional to the part of the routes i and j that overlap. This assumption can be shown to admit a desirable continuity in the definition of "alternative", avoiding the arbitrariness due to the enumeration of alternatives hitherto encountered.

The correlations introduced makes the evaluation of parameters more difficult, but do not necessarily complicate the prediction model.

Another point of consideration concerns the shape of the statistical distributions. If, for example, the A_k 's in formula (2:2) are assumed constant a_k and the X_{ik} 's are assumed independent and normal-distributed also the C_i 's will be normal and independent and the so called probit-model results. To simplify calculations, and usually with insignificant consequences, the probit-model may be approximated by a logit-model:

$$P_i = e^{-S \cdot C_i} / \sum_j e^{-S \cdot C_j} \quad \dots(2:3)$$

(s is a constant).

Finally, possible expressions for C_i must be considered. As criteria for the definition of the dimensions $k = 1, 2 \dots$ (in for example formula 2:2) should be considered (1) that the variables defined are suitable as input-parameters, (2) that, as an ensemble, they represent all the relevant information and (3) that they contain a minimum of redundancy. Rather than trying different variables on an ad-hoc basis, it should be possible to deduce reasonable assumptions regarding the set from, for example, an analysis of attitudinal data.

2.3 Results of a field survey

Method

In order to test the hypotheses mentioned and in an attempt to obtain some parameter values, a field survey was undertaken. This included 691 home interviews in Malmö and Lund, concerning route choice between specified origins and specified destinations. The interviews were divided among fourteen such origin-destination elements. Seven of these represented home-work trips, four home-city center and the remaining three home-other destinations.

The questionnaire included the following main items:

- route choice: most and second most preferred route
- trip characteristics: trip frequency, usual time of trip, usual purpose
- estimated route characteristics: estimated travel time and trip length, estimated differences between first and second choice (travel time, trip length and eleven semantic variables)
- reasons for route choice (preferences): fourteen attitudinal variables, using scaling-technique
- socio-economic data: six variables

A second survey, including 830 processed mail interviews with a similarly chosen sample but with a simpler questionnaire, was performed for validation purposes only.

Actual trip characteristics (13 variables) were measured. For travel times, a theoretical model was developed and calibrated through measurements. By help of a model for the cyclic flow variations, travel time could be predicted for any of the routes used any time of day.

The travel conditions represented in the sample may be characteristic for medium-size cities, but do not include heavily congested networks or trips of greater length than 5 - 6 km.

Influences of different types of variables

The correlations between route choice (actual characteristics for chosen route) and different types of "influencing" variables (estimated route differences, preferences, trip characteristics and socio-economic data) may be illustrated by the result of a canonical analyses. See FIG 1 below

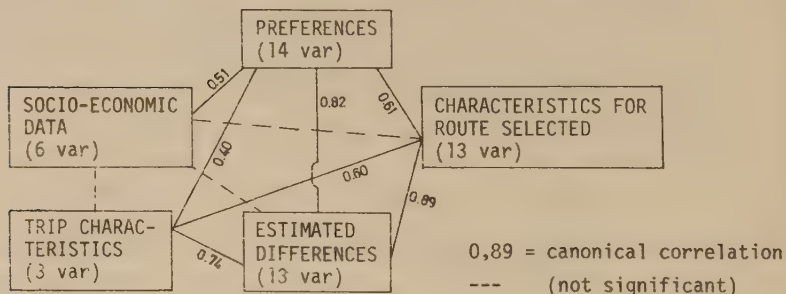


FIGURE 1. Canonical correlations between groups of variables used

The logical causal direction of the relations in FIG 1 is from left to right, but there is likely to be feed-backs from route choice to, for example, estimated differences between routes.

One point of interest with FIG 1 is that socio-economic variables have little influence on route choice. This agrees with most previous studies on the subject.

Stated reasons for choice

The attitudinal question, regarding reasons for choice of most preferred route, was included in the survey to give some indications as to which dimensions that are relevant in a description of route-choice.

A factor analysis of the fourteen variables, picked from an earlier similar study¹⁾ and from a pre-investigation with open-ended questions, gave the following result:

Factor name	Variable name	Factor load	o/o explained variance
"Smooth driving"	few stops	.73	17,5
	less congestion	.71	
	few signals	.54	
"Better road"	more lanes	.90	17,1
	fewer turns	.62	
"Shorter route"	less distance	.69	12,9
	less time	.60	
	less delay	.59	
"Few disturbances"	move relaxed	.77	8,5
	fewer pedestrians	.62	
	less risk	.59	
"Easier to find"	easier to find	.91	6,2
	fewer turns	.53	

TABLE 1. Result of factor analysis on attitudinal data

The factors proved to be relatively stable when dividing the sample, and a regression of the individual factors on route characteristics gave consistent results.

1) See Wachs, R "Relationships between driver attributes and driver and route characteristics" HRR 197 1967

Estimated route characteristics

Estimated differences in travel time and in other route characteristics, between first and second route, were investigated in relation to actual route characteristics and other variables.

The difference between actual and estimated travel time differences was found to have a bi-model distribution, indicating that people over- or underestimated the difference in favour of the route preferred.

After correction for this bias, the standard derivation of the difference estimated - actual travel time difference was calculated. This standard deviation, which is an important parameter in some traffic assignment methods, was found to increase with increasing travel time difference, although less than proportionally.

The estimated differences in other characteristics, as measured by a semantic scaling technique, were analyzed similarly.

Route choice model

The analysis briefly described admitted some conclusions concerning the choice of variables and variable definitions in the measure of resistance. Trying some different such measures, parameters were determined for the route-choice model (2:1), with the assumptions described.

In a first step, different binary models were calibrated by probit- or logit-technique using individual data and for each individual his first and second route. These individual data admit a more detailed analysis than what is possible after aggregation. It was shown, that the inclusion of more variables than just travel time and route length (both measured between the points a choice) meant significant improvements.

In a second step, data concerning each origin - destination element were aggregated and different multinomial models were fitted, corresponding to different hypotheses as mentioned in Ch 2.2. The best model obtained can be described as follows:

- the resistance C is defined for each link along a route (including the intersections), as a stochastic variable with a mean c , a standard deviation $s = k \cdot \sqrt{c}$ ($k = \text{constant}$) and a normal distribution
- the resistances of different links are independent and additive
- the choice probabilities are calculated by formula (2:1), observing the correlations introduced when two routes overlap (follows from the rules mentioned)
- the mean resistance should be calculated as

$$c = t (1 - 0,05kf + 0,10pk + 0,25gc) + \\ + 0,38 L + 1,25 t_s + 0,18n_s + 0,55 sv$$

(constant $k = 0,20$). The variables are defined in TABLE 2.

Variable	Name	Sort, definition
t	travel time	minutes
kf	number of lanes	-
pk	% of link length with < 50 m between parked cars	%
gc	pedestrians, bicycles along link	0 = none 3 = as in CBD
L	link length	km
t_s	stopped time	minutes
n_s	probability of stop	-
sv	number of turns > 45°	

TABLE 2. Definitions of route characteristics in model

The determination of the parameters was done by an iterative probit - technique. For prediction, a simulation technique is near at hand and has been applied in some existing assignment methods ¹⁾

1) See Burrell, J "Multiple route assignment and its applications to capacity restraint". Proc 4th Int Symp on the Theory of Traffic Flow, Karlsruhe 1968

Other hypotheses tested showed that the main improvement that may be possible, starting from the technique presently used in traffic assignment methods, concerns the inclusion of more aspects than travel time in the resistance measure.

The model described was also calibrated separately for different trip purposes. The differences obtained were barely significant.

Finally the parameters were again determined but using the material from the mail interviews, i.e. a different sample. A comparison of the results was encouraging.

Conclusions

There is reason to believe that existing traffic assignment methods give results with systematic bias of different kinds. It should be possible to eliminate some of this bias by use of a route choice model, developed along the lines indicated here and further developed to take into account changed travel conditions in congested flow. Some assignment methods, based on stochastic choice models similar to the ones tested in this study, are already in use and may be further improved by taking into account in the resistance measure more factors than just travel time and route length.

3. EXAMPLE 2: TRAFFIC FLOW AT UNCONTROLLED INTERSECTIONS

3.1 Background and purpose

In most existing capacity manuals, intersections without signal control are treated in a rather crude fashion. Nevertheless, of the total delays suffered in city traffic a major part amounts from such intersections.

One reason for this state of affairs is the lack of empirical data. Not until the last ten or so years has it been possible, with the help of theoretical models, to predict capacity, delay and similar measures - of - efficiency with some accuracy. With the help of such models, the need for empirical data can be reduced to feasible dimensions.

The study described here is part of a major project, undertaken by the National Road Administration with the purpose of reviewing capacity calculation procedures for different types of road traffic facilities. The work done on uncontrolled intersections fall in three major parts: analysis of existing results and practices, a field study with the explicit purpose of measuring critical headways and finally the development of new procedures. In this report is mainly contained some results of the first two of these parts.

For a more comprehensive discussion of the study the reader is referred to APPENDIX 2.

3.2 Theoretical models

Definition

Consider the simple intersection in FIG 2, between two one-way, one-lane roads one of which has priority over the other.

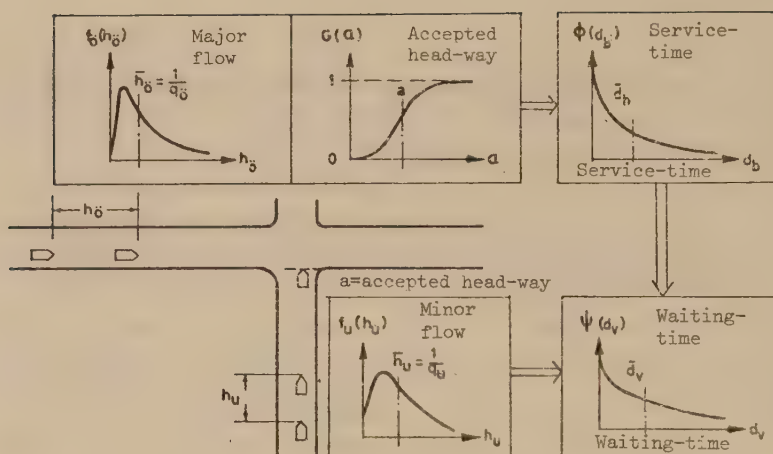


FIGURE 2. Definition of delay and its different components

Define delay for a minor road vehicle as the extra travel time required through the intersection (major road vehicles are assumed to suffer no delay). For this total delay, different components may be defined as follows:

- d_f running delay (due to retardation and acceleration only)
- $d_v = d_b + d_k$ waiting-time (time standing still, including move-up times within queue)
- d_b service-time (time in first position)
- d_k queueing-time (time in queue until first position is reached)

Of these components, only the last three shall be subjected to theoretical treatment here, while running delay is estimated better by use of empirical results.

The capacity of the minor road approach is defined as the maximum flow that can be maintained as an average for a long period of time, at a constant average demand and for a certain constant major road flow.

The queue-length in the minor road approach is defined as the number of cars already waiting (including the one being served) when a new minor road vehicle arrives.

The extension of the definitions above to more realistic intersections, with several lanes, is straight-forward and is omitted here. Also, the "minor-road approach" in FIGURE 2 might be a separate lane for left-turning traffic on a major road.

Survey

Existing theoretical models for calculating delay and capacity, for a minor road approach at an uncontrolled intersection, primarily consider the basic interaction between two flows as in FIG2. In terms of queueing theory, the principal procedure may be described by FIGURE 3.

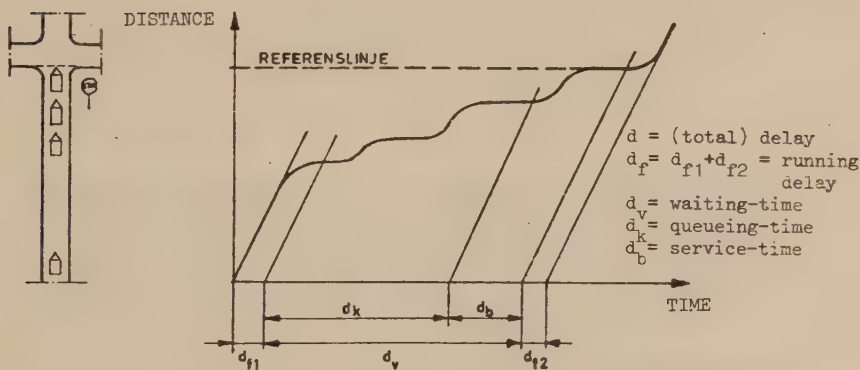


FIGURE 3. Principal procedure for the calculation of delay and capacity by queueing theory.

The scheme can be explained as follows:

- a. The function $f_0(h_0)$ describes the frequencies of head-ways of different size h_0 in the major road flow.
- b. The function $G(\alpha)$ expresses the probability that a head-way ("gap") of size $h_0 = \alpha$ will be accepted by a waiting minor-road vehicle (i.e. that the vehicle will cross)
- c. The statistical distribution (d_b) of the service-time d_b mainly depends on the frequency of head-ways (a) and the probability that these are accepted (b), and little on what happens farther back in the queue.

Particularly, the capacity of the approach K (vehicles per second) must approximately equal the reciprocal of the mean service-time $\bar{d}_b$ (seconds per vehicle)

- d. The function $f_u(h_u)$ describes the frequencies of head-ways of different size h_u in the arriving minor road flow.
- e. The statistical distribution (d_v) of the waiting-time d_v mainly depends on the frequency of different intervals between departures (c) and on the frequency of different intervals between arrivals (d). The same applies to the distribution of queue length.

Input parameters in the model in FIG 3 are the head-way distributions in the two flows, with means $1/q_0$ and $1/q_u$, and the distribution of the accepted head-ways. For practical calculations of delay the flows q_0 and q_u are supposedly known and some assumption made concerning the distributions round the means.

The gap-acceptance distribution is often represented by a step-function, i.e. some "critical gap" $\underline{a}$ is defined. The value of this "critical gap" will greatly depend on variables describing the intersection design and control. Also, the critical gap will be the main descriptor of the influence of such variables.

With the model described, which includes some approximations that will be discussed presently, capacity and mean waiting-time can be estimated. Below, the boxes d and e in FIG 3 for the basic interaction shall be

treated first. After that, possible generalizations to more realistic intersections, with several superimposed interactions, will be considered briefly.

Service-time and capacity

The service-time d_b will (on the average) be slightly smaller during conditions without queue than when there is a queue. In the second case, service-time is counted from the moment the preceding vehicle leaves the approach. From this moment, it will take the new queue-leader some time to reach the intersection. Call this time the move-up time, a_f .

Consider first the case without queue, i.e. when traffic flow in one or both approaches is very low. If all drivers are assumed to require one and the same critical head-way a (s), and if the head-ways in the major road flow are assumed exponential, the average service-time can be derived as

$$\bar{d}_b^0 = \frac{1}{q_0} (e^{aq_0} - aq_0 - 1) \quad \dots(3:1)$$

where q_0 (v/s) is the major road flow.

In the second case, assuming a constant queue of vehicles in the minor road approach, it must be considered that more than one vehicle may pass in one and the same head-way. A common hypothesis is that two vehicles need a head-way of at least $(a + a_f)$ to pass, three $(a + 2a_f)$ etcetera, where a_f is the move-up time as defined.

With this assumption, the capacity of the minor road approach is easily computed. For exponential head-ways is obtained

$$\begin{aligned} K &= \sum_{i=1}^{\infty} i \cdot q_0 (e^{-q_0(a+(i-1)a_f)} - e^{-q_0(a+ia_f)}) = \\ &= q_0 e^{-aq_0} / (1 - e^{-a_f q_0}) \quad \dots(3:2) \end{aligned}$$

This gives the following mean service time during queueing conditions:

$$\bar{d}_b^k = 1/K = \frac{1}{q_0} (e^{aq_0} - e^{(a-a_f)q_0}) \quad \dots(3:3)$$

The formulas (3:1) - (3:3) are well known and may also be modified for other assumptions regarding head-way distribution and gap-acceptance.¹⁾ Formula (3:2) for capacity is often used in practical applications and has shown good agreement with empirical results.²⁾

The expressions (3:1) and (3:3) for service-time generally differ with an amount that is less than the move-up time a_f . For load-factors in between zero (= no queue) and one (=constant queue), mean service-time $\bar{d}_b$ will assume a value in between $\bar{d}_b^0$ and $\bar{d}_b^k$.

Waiting-time and queue-length

If head-ways in the arriving minor-road flow are independent and exponential, and if service-times for successive vehicles are independent, a well-known result from queueing theory gives the mean waiting-time as

$$\bar{d}_v = \bar{d}_b \left(1 + \frac{q_u \bar{d}_b}{1 - q_u \bar{d}_b} \cdot \frac{1+c^2}{2} \right) \quad \dots(3:3)$$

where

$\bar{d}_v$ = mean waiting-time

$\bar{d}_b$ = mean service-time

q_u = minor road flow

c = coefficient of variation for the distribution of service-times (standard deviation / mean)

(From the so-called Pollaczek's formula). The standard deviation of the service-times can be derived for the same conditions as the mean, and thus the coefficient of variation c . In many cases of interest, c is very nearly one which indicates a nearly exponential distribution

1) For references see APPENDIX 2

2) See Owens, D "Flow measurements at a number of uncontrolled T-junctions", RRL Report LR 171 1969.

of service-times. This corresponds to the simple $M / M / 1$ queueing system ($M = \text{Markov}$).

According to formula 3:3 the waiting-time tends to infinity as the product $q_u \bar{d}_b$ approaches 1,0. This product is the load factor B .

The assumptions that lead to formula (3:3) are not entirely satisfactory. The influence of other arrival processes than the one specified can usually be neglected for practical applications. However, service-times are not entirely independent, due to the difference between queueing and non-queueing vehicles, which usually leads to an over-estimation of waiting-time.

More exact formulas have been derived without this assumption of independent service-times¹⁾, but do not admit generalizations to the same extent as formula (3:3).

If mean waiting-time has been calculated the average number of cars in queue at a new arrival can easily be obtained (or vice versa), as

$$\bar{N} = \bar{d}_v q_u \quad \dots (3:4)$$

Superimposed interactions

So far, only the basic interactions between two streams of vehicles has been considered. In a road intersection, as in FIG 4, several such interactions are superimposed and have to be treated jointly.

1) See Tanner, J "A theoretical analysis of delays at an uncontrolled intersection", Biometrika Vol 49 pp 163 - 170 1962.

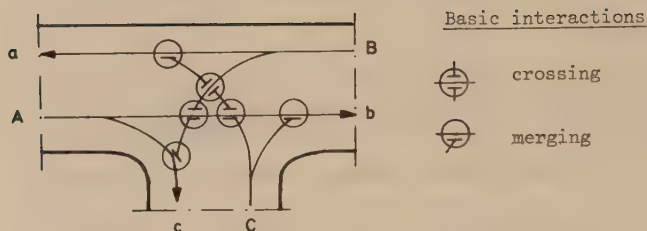


FIGURE 4. Interactions in a T-junction.

The problems that arise can be classified in three categories:

- (i) a non-priority stream, such as C_a in FIG 4, has to give way to more than one priority stream (Ab , Bc , Ba).
- (ii) a stream, such as Bc in FIG 4, has to give way to some streams (Ab , Ac) but has priority to others (Ca).
- (iii) several streams, with different priority conditions, use the same lane (Cb , Ca).

The first type of problems has been subjected to theoretical treatments and can be solved analytically. Consider for example the stream Ca in FIG 4. If the head-ways in the three streams Ab , Ba and Bc are all independent and exponential, the probability of a simultaneous head-way bigger than a critical value a will be

$$e^{-q_{Ab} \cdot a} \cdot e^{-q_{Ba} \cdot a} \cdot e^{-q_{Bc} \cdot a} = e^{-(q_{Ab} + q_{Ba} + q_{Bc})a} \quad \text{..(3:4)}$$

That is, those flows which have priority in relation to a non-priority stream are simply added. For other head-way distributions, the results are more complex.

The second type of problems is more complex. In FIG 4, a simultaneous head-way in the streams Ab and Ba can not be utilized by a Ca vehicle

as long as there is at least one vehicle of type Bc waiting. The share of time when this is the case can be shown to equal B_B , the load factor for approach B. A common procedure¹⁾ is then to reduce the capacity (increase mean service-time) for Ca-vehicles with the factor $(1-B_B)$, i.e. the probability that approach B is empty.

The third type of problems is again easier, when using a queueing model like formula (3:3). If means and standard-deviations of the service-times are calculated separately for the flows Ca and Cb in FIG 4, they can be averaged to give a joint mean and a joint standard deviation. These statistics may then be used to estimate the waiting-time by formula (3:3). Particularly, the capacity of the approach C can be approximated by the joint mean service-time.

Conclusions

The possibility of using theoretically developed formulas, based on the concept of accepted head-ways, for practical calculations of capacity, waiting-time and queue-length seem promising. Much work remains however, considering the evaluation of different approximations, the determining of parameter values related to gap-acceptance and to head-way distributions, the inclusion of effects of such constructions as short lanes or pedestrian crossings and, finally, the validation of the methods.

1) See Harders, J "Leistungsfähigkeit nicht signalgeregelter städtischen Verkehrsknoten", Str.bau u Str.verkehrstechnik, Heft 76 1968.

3.3 Measurements of critical head-ways

Definition

Several different distribution functions can be defined for accepted head-ways, depending on the following factors:

- (i) type of choice situation studied. Most common is the distinction between
 - lag = remainder of a head-way in the major road flow, counted from the arrival of a minor road vehicle
 - gap = head-way between successive vehicles in major road flow
- (ii) assumptions made concerning the cause of the distribution around the mean, for example (1) mainly due to differences between drivers or (2) mainly due to "inconsistent" individual behaviour
- (iii) points of reference for defining head-ways (potential collision points, boundaries of intersection)

Further, several methods exist for the calculation of a "critical" head-way from a head-way distribution, and for the correction of bias when considering gaps (drivers with high critical gaps reject more gaps than others do).

In this study, both gaps and lags were measured, as well as the move-up time previously defined. All head-ways were related to the potential collision points, for the streams involved, in order to maintain consistency with the theoretical models described. Critical head-ways were estimated as means of assumed log-normal acceptance functions, using probit-analysis and correction for bias.

Sample

Eighteen intersections were selected for the study, fifteen in urban areas of various sizes and three on rural locations.

1) See Miller, A "Nine estimators of gap-acceptance parameters". Proc 5th Int Symp on the Theory of Traffic Flow and Transportation, Berkeley 1971.

The sample should be representative for the most common types of intersections. It should also, as far as possible, admit pair-wise comparisons concerning the effects of the most important influencing variables (stop or give-way sign, speed limit, location, number of lanes, cross-section of major road, load factor).

In order to simplify such comparisons, only intersections with relatively few disturbances of any kind were included.

Method

An automatic data collection technique, based on detectors and data-logging, was developed and used. In the minor road approaches studied were installed presence-type magnetic loop detectors, combined with pneumatic tube detectors in a logic circuit in an attempt to minimize errors. Usually two detectors were used per lane, in order to detect both the first and the second vehicle in a queue. Manual functions were used only to indicate directions (choice of exit from the intersection) and different types of disturbances in the flows.

The effective time of observation in each intersection ranges from 6 to 10 hours, cut into periods of 20 - 30 minutes and distributed over peak and off-peak situations.

For the processing, a computer program was written that covered all the types of intersections studied. In some introductory rounds of processing, the data was used to determine the time constants needed for the definitions of gaps and lags, and to what extent different major road flows should be counted in the conflicts with different minor road flows.

Results

The critical head-ways measured are listed in TABLE 3 below, together with obtained values of the move-up time (a_f). Gaps and lags have been pooled in the table.

LOCATION	CONTROL s=stop g=give way	SPEED LIMIT (km/h)	NUMBER OF LEGS	LANES MAJOR ROAD	CRITICAL HEAD-WAY (s)			a_f (s)
					Right turn	Straight	Left turn	
Malmö	g	50	3	4	5,0	-	6,3	3,0
"	s	50	4	6	5,1	6,0	6,1	3,5
Lomma	s	50	3	2	5,5	-	6,0	- ¹⁾
Lund	s	50	4	2	5,6	- ¹⁾	5,9	3,6
"	g	50	4	2	5,9	5,1	- ¹⁾	- ¹⁾
Helsingborg	g	50	3	2-4	4,7	-	5,6	3,0
v15 - v23	g	90	2	2	3,5	-	-	- ¹⁾
v107 - v109	s	90	4	2	6,0	7,0	7,4	- ¹⁾
Hässleholm	g	50	4	2	4,6	6,0	6,0	3,1
Stockholm	g	50	3	4	4,0	-	4,9	2,6
"	g	50	3	4	4,3	- ¹⁾	5,1	2,9
"	g	50	3	4	3,9	-	4,5	2,2
Eskilstuna	s	50	4	2	6,5	6,7	6,6	4,0
E4 - v218	s	90	4	4	6,2	6,9	7,5	- ¹⁾
Norrköping	s	70	4	6	3,7	5,3	5,2	3,5
"	v	50	4	4	3,3	4,7	5,3	2,9
Västerås	s	70	4	4	6,7	-	7,3	4,2
"	v	70	3	4	6,7	-	6,4	4,1

1) Few observations

TABLE 3. Measured critical head-ways for 18 intersections

The confidence limits for the individual critical head-ways in the table vary between $\pm 0,1$ s and $\pm 1,0$ s. The confidence limits for the move-up times are normally $\pm (0,15 - 0,30)$ s.

For some of the intersections, the critical head-ways were measured also for left turning traffic from one of the major road approaches. The results range from 4,6 - 6,1 s, the highest value concerning a high-way with 90 km/h speed limit. Other differences are not significant.

A result of some consequences, observed during the preliminary processing, was that the definition of which major road flows that should define head-ways has a rather big influence on critical head-ways. As criteria for the inclusion or exclusion of a certain major road flow in a certain lane, and with respect to a certain minor road flow, was used the variance of the acceptance distribution - a smaller variance meaning that the head-ways defined are more relevant for describing the acceptance behaviour. One main conclusion drawn is that in a merging conflict, the minor road vehicle is little affected by approaching vehicles in other lanes than the merging lane.

Different components of delay were measured and evaluated for six intersections. For the mean waiting-time, measured as the difference in travel time through the intersection for all vehicles and vehicles that arrived without queue and accepted the first head-way (lag) respectively, results were compared with theoretical calculations as described in 3.2. The following regression line was obtained for 55 observations (20 - 30 minutes each):

$$\bar{d}_{vt} = - 0,61 + 1,18 \bar{d}_{vm} \quad (R^2 = 0,79)$$

where $\bar{d}_{vt}$ is theoretical (through using actual flows and measured critical head-ways) and $\bar{d}_{vm}$ measured average waiting-time.

Conclusions

The following are the main conclusions from the measurements:

- the general level of observed critical head-ways agree well

with previous studies¹⁾

- on the whole, the influence of the most important variables on the critical head-ways were as could be expected from previous studies: the critical head-ways are higher at stop than at yield-signs, higher with increasing speeds (speed limits) on the major road and lower in big cities (Stockholm).
- one point of difference may be the influence of width of the major road. Previously reported results, mentioning substantially higher critical head-ways on four - lane than on two-lane roads, could not be confirmed.
- the move-up time is related to the critical head-way and increases thus with increasing critical head-way. Rather big deviations were observed in some intersections, however, that could not be explained but that could be random.
- care should be taken when defining head-ways with respect to different interacting flows. More research will be needed here to substantiate the findings in this study.

1) See references in APPENDIX 2.

